A. Forces on drop connector bottom portion:

$P := -870 \cdot lbf$ Applied load

$R1$ & $R2$ Reaction forces at bolted connections

$P + R1 + R2 = 0$ Sum of forces in vertical direction

$R2 = -P - R1$

$A := 26.784 \cdot in$ $B := 20.517 \cdot in$ $C := 6.267 \cdot in$ Distances of forces from right edge

$P \cdot A + R1 \cdot B + R2 \cdot C = 0$ Sum of moments

$P \cdot A + R1 \cdot B + (-P - R1) \cdot C = 0$

$$R1 := \frac{-P \cdot (A - C)}{(B - C)} = 1.253 \times 10^{3} \cdot lbf$$

$R2 := -P - R1 = -383 \cdot lbf$

B. Allowable shear on bolts (Section J3.6) for the bottom portion of the connector:

$F_{nv} := 48 \cdot ksi$ nominal shear stress from Table J3.2

$n := 4$ number of bolts $d := 0.5 \cdot in$ nominal bolt diameter

$A_b := n \cdot \left(\frac{\pi}{4} \cdot d^2\right) = 0.785 \cdot in^2$ area of bolts

$R_n := F_{nv} \cdot A_b = 37.699 \times 10^{3} \cdot lbf$ nominal shear strength of bolted connection (J3-1)

$\Omega := 2.00$ ASD safety factor $R_a := \frac{R_n}{\Omega} = 18.85 \times 10^{3}\, lbf$ allowable shear strength

$R1 < R_a$ therefore, bolts have adequate shear strength

C. Bearing strength at bolt holes (Section J3.10):

$F_u := 58 \cdot ksi$ minimum tensile strength of the material

$L_c := 0.735 \cdot in$ clear distance from edge of hole to edge of material

$t := 0.25 \cdot in$ thickness of material

$R_n := 1.0 \cdot L_c \cdot t \cdot F_u = 10.657 \times 10^3\,lbf$ nominal bearing strength, Equation (J3-6c)

$\Omega := 2.00$ ASD safety factor

$R_a := \frac{R_n}{\Omega} = 5.329 \times 10^3\,lbf$ allowable bearing strength

$R1 < R_a$ therefore, bolt holes have adequate bearing strength

D. Strength of connecting elements in tension (Section J4.1):

Calculated for the vertical plate of the bottom portion of the drop connector.

Allowable strength is the lesser of tensile yielding or tensile rupture.

$A_g := 19.83 \cdot in \cdot 0.25 \cdot in = 4.957\,in^2$ Gross area of element section

$F_y := 36 \cdot ksi$ Yield stress of A36 steel

$R_n := F_y \cdot A_g = 178.470 \times 10^3\,lbf$ Nominal tensile yielding strength

$\Omega := 1.67$ ASD safety factor

$R_a := \frac{R_n}{\Omega} = 106.868 \times 10^3\,lbf$ Allowable tensile yield strength

$A_e := 19.83 \cdot in \cdot .25 \cdot in - 2 \cdot (4.096 \cdot in \cdot .25 \cdot in) = 2.909\,in^2$ Effective area of element section

$F_u := 58 \cdot ksi$ Ultimate tensile strength of A36

$R_n := F_u \cdot A_e = 168.751 \times 10^3\,lbf$ Nominal tensile rupture strength

$\Omega := 2.00$ ASD safety factor

$R_a := \frac{R_n}{\Omega} = 84.375 \times 10^3\,lbf$ Allowable tensile rupture strength

R1 is less than both the allowable tensile yield and tensile rupture strengths; therefore strength of element is adequate.

E. Strength of weld joint in bottom portion (Section J2.4, Table J2.5)

(Weld is 1/4" groove weld in 5 places, each 2" long, assumed to be partial-penetration)

Base metal strength

$A_g := 10 \cdot in \cdot 0.25 \cdot in = 2.5\, in^2$ Gross area of base metal

$F_y := 36 \cdot ksi$ Yield stress of A36 steel

$R_n := F_y \cdot A_g = 90.000 \times 10^3\, lbf$ Nominal tensile yielding strength

$\Omega := 1.67$ ASD safety factor

$R_a := \frac{R_n}{\Omega} = 53.892 \times 10^3\, lbf$ Allowable tensile yield strength

$R1 < R_a$ therefore, base metal strength is adequate

Weld strength

$A_w := 10 \cdot in \cdot .25 \cdot in = 2.5\, in^2$

$F_{EXX} := 60 \cdot ksi$ Ultimate tensile strength of filler metal

$F_w := 0.60 \cdot F_{EXX} = 3.6 \times 10^4\, psi$ Nominal strength of filler metal

$R_n := F_w \cdot A_w = 90.000 \times 10^3\, lbf$

$\Omega := 1.88$

$R_a := \frac{R_n}{\Omega} = 47.872 \times 10^3\, lbf$

$R1 < R_a$ therefore, weld strength is adequate

F. Forces on drop connector top portion:

$P := -1253 \cdot lbf$ Applied load

$R1$ & $R2$ Horizontal reaction forces at bolted connection

$R3$ Vertical reaction force at bolted connection

$P + R3 = 0$ Sum of forces in vertical direction

$R3 := -P = 1.253 \times 10^3\, lbf$

$R1 + R2 = 0$ Sum of forces in horizontal direction

$R1 = -R2$

$A := 7 \cdot in$ $B := 4.5 \cdot in$ Distances from common point to sum moments

$P \cdot A + R2 \cdot B = 0$ Sum of moments

$R2 := \frac{-P \cdot A}{B} = 1.949 \times 10^3 \cdot lbf$

$R1 := -R2 = -1.949 \times 10^3\, lbf$

G. Allowable shear on bolts (Section J3.6) for the top portion of the connector:

$F_{nv} := 48 \cdot ksi$ nominal shear stress from Table J3.2

$n := 2$ number of bolts $d := 0.5 \cdot in$ nominal bolt diameter

$A_b := n \cdot \left(\frac{\pi}{4} \cdot d^2 \right) = 0.393 \cdot in^2$ area of bolts

$R_n := F_{nv} \cdot A_b = 18.85 \times 10^3 \cdot lbf$ nominal shear strength of bolted connection (J3-1)

$\Omega := 2.00$ ASD safety factor $R_a := \frac{R_n}{\Omega} = 9.425 \times 10^3\, lbf$ allowable shear strength

$R3 < R_a$ therefore, the two bolts in shear are adequately strong

H. Allowable tension on bolts (Section J3.6) for the top portion of the connector:

$F_{nt} := 90 \cdot ksi$ nominal tensile stress from Table J3.2

$n := 2$ number of bolts $d := 0.5 \cdot in$ nominal bolt diameter

$A_b := n \cdot \left(\frac{\pi}{4} \cdot d^2 \right) = 0.393 \cdot in^2$ area of bolts

$R_n := F_{nt} \cdot A_b = 35.343 \times 10^3 \cdot lbf$ nominal shear strength of bolted connection (J3-1)

$\Omega := 2.00$ ASD safety factor $R_a := \frac{R_n}{\Omega} = 17.671 \times 10^3\, lbf$ allowable tensile strength

$R2 < R_a$ therefore, the two bolts that are in tension are adequately strong

I. Weld shear strength in top portion of connector:

(From photo and measurements, appears to be 1/4" fillet, 5" in length, both sides)

$throat := \frac{0.25 \cdot in}{\sqrt{2}} = 0.177\,in$ throat of 1/4" fillet weld

$A_w := (throat \cdot 5 \cdot in) \cdot 2 = 1.768\,in^2$ area of weld

$F_{EXX} := 60 \cdot ksi$ Ultimate tensile strength of filler metal

$F_w := 0.60 \cdot F_{EXX} = 36.000 \times 10^3\,psi$ Nominal tensile strength of filler metal

$R_n := F_w \cdot A_w = 63.640 \times 10^3\,lbf$ Nominal strength of weld

$\Omega := 2.00$ ASD Safety Factor

$R_a := \frac{R_n}{\Omega} = 31.820 \times 10^3\,lbf$ Allowable shear strength of fillet weld

$R3 < R_a$ therefore, fillet weld has adequate shear strength

J. Compressive strength for flexural buckling of members without slender elements (Section E3)

This calculation evaluates the strength of the W6x15 columns in the service beam support structure.

$b_f := 5.99 \cdot in$ $t_f := 0.260 \cdot in$ flange dimensions

$\lambda := \frac{b_f}{2 \cdot t_f} = 11.519$ width - thickness ratio of member

$E := 29000 \cdot ksi$ $F_y := 36 \cdot ksi$ material properties of A36 steel

$\lambda_p := 0.38 \cdot \sqrt{\frac{E}{F_y}} = 10.785$ limit for compact sections (Table B4.1)

$\lambda_r := 1.0 \cdot \sqrt{\frac{E}{F_y}} = 28.382$ limit for noncompact sections (Table B4.1)

$\lambda_p < \lambda < \lambda_r$ therefore section is considered noncompact

$k := 1$ effective length factor determined in accordance with Section C2

$L := 16 \cdot ft + 4 \cdot in$ laterally unbraced length of member

$r := 1.45 \cdot in$ governing radius of gyration, taken from Table 1-1

$\frac{k \cdot L}{r} = 135.172$ column slenderness ratio

$4.71 \cdot \sqrt{\frac{E}{F_y}} = 133.681$ when slenderness ratio exceeds this value, Equation E3-3 applies

$F_e := \frac{\pi^2 \cdot E}{\left(\frac{k \cdot L}{r}\right)^2} = 15.665 \cdot ksi$ first determine elastic critical buckling stress from Equation E3-4

$F_{cr} := 0.877 \cdot F_e = 13.738 \cdot ksi$ flexural buckling stress Equation E3-3

$A_g := 4.43 \cdot in^2$ gross cross sectional area of member from Table 1-1

$P_n := F_{cr} \cdot A_g = 60.859 \times 10^3 \, lbf$ nominal compressive strength

$\Omega_c := 1.67$ ASD Safety Factor

$\frac{P_n}{\Omega_c} = 3.644 \times 10^4 \, lbf$ allowable strength of column

$P := 870 \cdot lbf + 420 \cdot lbf = 1.29 \times 10^3 \, lbf$ applied load to column

$P < \frac{P_n}{\Omega_c}$ therefore, column has adequate strength

K. Doubly symmetric compact I-shaped members bent about their major axis (Section F2)

Suggested beam size of W6x15 was found to be inadequate to support the applied load. A W8x21 beam was chosen to replace it.

$b_f := 5.27 \cdot in$ $t_f := 0.40 \cdot in$ dimensions of W8x21 flange

$\lambda := \dfrac{b_f}{2 \cdot t_f} = 6.587$ width - thickness ratio

$\lambda < \lambda_p$ therefore section is compact according to Table B4.1

The nominal flexural strength shall be the lower value according to the limit states of yielding and lateral-torsional buckling.

$L := 20 \cdot ft$ length of beam

$P := 870 \cdot lbf$ applied point load on beam

$M := \dfrac{P \cdot L}{4} + 1050 \cdot ft \cdot lbf = 5.4 \times 10^3 \cdot ft \cdot lbf$ applied moment on beam from point load and weight of beam

Section F2.1 Yielding:

$Z_x := 20.4 \cdot in^3$ plastic section modulus about x-axis, from Table 1-1

$F_y = 36 \cdot ksi$ minimum yield stress of A36 steel

$M_p := F_y \cdot Z_x = 6.12 \times 10^4 \cdot ft \cdot lbf$ plastic moment

$M_n := M_p$ nominal flexural strength

$\Omega_b := 1.67$ ASD Safety Factor

$M_a := \dfrac{M_n}{\Omega_b} = 3.665 \times 10^4 \cdot ft \cdot lbf$

$M < M_a$ therefore beam does not yield

Section F2.2 Lateral-torsional buckling (LTB):

$L_b := L = 20 \cdot ft$ unbraced length of beam

$r := 1.26 \cdot in$ radius of gyration in x-axis, from Table 1-1

$$L_p := 1.76 \cdot r \cdot \sqrt{\frac{E}{F_y}} = 5.245 \cdot ft$$ lower limit for LTB

The next row of parameters are needed to determine Lr, and are taken from Table 1-1

$S_x := 18.2 \cdot in^3 \quad r_{ts} := 1.46 \cdot in \quad J := 0.282 \cdot in^4 \quad h_o := 7.88 \cdot in$

$c := 1$ from (F2-8a)

$$L_r := 1.95 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{\frac{J \cdot c}{S_x \cdot h_o}} \cdot \sqrt{1 + \sqrt{1 + 6.76 \cdot \left(\frac{0.7 \cdot F_y}{E} \cdot \frac{S_x \cdot h_o}{J \cdot c}\right)^2}} = 20.959 \cdot ft$$

$L_p < L_b < L_r$ therefore nominal flexural strength for LTB determined by Equation F2-2

$C_b := 1$ permitted to be conservatively taken as 1.0 for all cases

$$M_n := C_b \cdot \left[M_p - \left(M_p - 0.7 \cdot F_y \cdot S_x\right) \cdot \left(\frac{L - L_p}{L_r - L_p}\right)\right] = 3.962 \times 10^4 \cdot ft \cdot lbf$$

$$M_a := \frac{M_n}{\Omega_b} = 2.373 \times 10^4 \cdot ft \cdot lbf$$

$M < M_a$ therefore beam is strong enough to prevent LTB